

Chapter 3 Geotechnical Investigation

3-1. Planning the Investigation

a. Purpose. The purpose of the geotechnical investigation for wall design is to identify the type and distribution of foundation materials, to identify sources and characteristics of backfill materials, and to determine material parameters for use in design/analyses. Specifically, the information obtained will be used to select the type and depth of wall, design the sheet pile wall system, estimate earth pressures, locate the ground-water level, estimate settlements, and identify possible construction problems. For flood walls, foundation underseepage conditions must also be assessed. Detailed information regarding subsurface exploration techniques may be found in EM 1110-1-1804 and EM 1110-2-1907.

b. Review of existing information. The first step in an investigational program is to review existing data so that the program can be tailored to confirm and extend the existing knowledge of subsurface conditions. EM 1110-1-1804 provides a detailed listing of possible data sources; important sources include aerial photographs, geologic maps, surficial soil maps, and logs from previous borings. In the case of floodwalls, study of old topographic maps can provide information on past riverbank or shore geometry and identify likely fill areas.

c. Coordination. The geotechnical investigation program should be laid out by a geotechnical engineer familiar with the project and the design of sheet pile walls. The exploration program should be coordinated with an engineering geologist and/or geologist familiar with the geology of the area.

3-2. Subsurface Exploration and Site Characterization

a. Reconnaissance phase and feasibility phase exploration: Where possible, exploration programs should be accomplished in phases so that information obtained in each phase may be used advantageously in planning later phases. The results of each phase are used to "characterize" the site deposits for analysis and design by developing idealized material profiles and assigning material properties. For long, linear structures like floodwalls, geophysical methods such as seismic and resistivity techniques often provide an ability to

rapidly define general conditions at modest cost. In alluvial flood plains, aerial photograph studies can often locate recent channel filling or other potential problem areas. A moderate number of borings should be obtained at the same time to refine the site characterization and to "calibrate" geophysical findings. Borings should extend deep enough to sample any materials which may affect wall performance; a depth of five times the exposed wall height below the ground surface can be considered a minimum "rule of thumb." For floodwalls atop a levee, the exploration program must be sufficient not only to evaluate and design the sheet pile wall system but also assess the stability of the overall levee system. For floodwalls where underseepage is of concern, a sufficient number of the borings should extend deep enough to establish the thickness of any pervious strata. The spacing of borings depends on the geology of the area and may vary from site to site. Boring spacing should be selected to intersect distinct geological characteristics of the project.

b. Preconstruction engineering and design phase. During this phase, explorations are conducted to develop detailed material profiles and quantification of material parameters. The number of borings should typically be two to five times the number of preliminary borings. No exact spacing is recommended, as the boring layout should be controlled by the geologic conditions and the characteristics of the proposed structure. Based on the preliminary site characterization, borings should be situated to confirm the location of significant changes in subsurface conditions as well as to confirm the continuity of apparently consistent subsurface conditions. At this time, undisturbed samples should be obtained for laboratory testing and/or in situ tests should be performed.

c. Construction general phase. In some cases, additional exploration phases may be useful to resolve questions arising during detailed design to provide more detailed information to bidders in the plans and specifications, subsequent to construction, or to support claims and modifications.

3-3. Testing of Foundation Materials

a. General. Procedures for testing soils are described in EM 1110-2-1906. Procedures for testing rock specimens are described in the *Rock Testing Handbook* (U.S. Army Engineer Waterways Experiment Station (WES) 1980). Much of the discussion on use of laboratory tests in EM 1110-1-1804 and EM 1110-2-1913 also applies to sheet pile wall design.

Classification and index tests (water content, Atterberg limits, grain size) should be performed on most or all samples and shear tests should be performed on selected representative undisturbed samples. Where settlement of fine-grain foundation materials is of concern, consolidation tests should also be performed. The strength parameters ϕ and c are not intrinsic material properties but rather are parameters that depend on the applied stresses, the degree of consolidation under those stresses, and the drainage conditions during shear. Consequently, their values must be based on laboratory tests that appropriately model these conditions as expected in the field.

b. Coarse-grain materials (cohesionless). Coarse-grain materials such as sands, gravels, and nonplastic silts are sufficiently pervious that excess pore pressures do not develop when stress conditions are changed. Their shear strength is characterized by the angle of internal friction (ϕ) determined from consolidated, drained (S or CD) tests. Failure envelopes plotted in terms of total or effective stresses are the same, and typically exhibit a zero c value and a ϕ value in the range of 25 to 45 degrees. The value of ϕ for coarse-grain soils varies depending predominately on the particle shape, gradation, and relative density. Because of the difficulty of obtaining undisturbed samples of coarse-grain soils, the ϕ value is usually inferred from in situ tests or conservatively assumed based on material type.

(1) Table 3-1 shows approximate relationships between the relative density, standard penetration resistance (SPT), angle of internal friction, and unit weight of granular soils. Figure 3-1 shows another correlation between ϕ , relative density, and unit weight for various types of coarse-grain soils. Where site-specific correlations are desired for important structures, laboratory tests may be performed on samples recompacted to simulate field density.

(2) The wall friction angle, δ , is usually expressed as a fraction of the angle of internal friction, ϕ . Table 3-2 shows the smallest ratios between δ and ϕ determined in an extensive series of tests by Potyondy (1961). Table 3-3 shows angle of wall friction for various soils against steel and concrete sheet pile walls.

c. Fine-grain materials (cohesive soils). The shear strength of fine-grain materials, such as clays and plastic silts, is considerably more complex than coarse-grain soils because of their significantly lower permeability,

higher void ratios, and the interaction between the pore water and the soil particles.

(1) Fine-grain soils subjected to stress changes develop excess (either positive or negative) pore pressures because their low permeability precludes an instantaneous water content change, an apparent $\phi = 0$ condition in terms of total stresses. Thus, their behavior is time dependent due to their low permeability, resulting in different behavior under short-term (undrained) and long-term (drained) loading conditions. The condition of $\phi = 0$ occurs only in normally consolidated soils. Overconsolidated clays "remember" the past effective stress and exhibit the shear strength corresponding to a stress level closer to the preconsolidation pressure rather than the current stress; at higher stresses, above the preconsolidation pressure, they behave like normally consolidated clays.

(2) The second factor, higher void ratio, generally means lower shear strength (and more difficult designs). But in addition, it creates other problems. In some (sensitive) clays the loose structure of the clay may be disturbed by construction operations leading to a much lower strength and even a liquid state.

(3) The third factor, the interaction between clay particles and water (at microscopic scale), is the main cause of the "different" behavior of clays. The first two factors, in fact, can be attributed to this (Lambe and Whitman 1969). Other aspects of "peculiar" clay behavior, such as sensitivity, swelling (expansive soils), and low, effective- ϕ angles are also explainable by this factor.

(4) In practice, the overall effects of these factors are indirectly expressed with the index properties such as LL (liquid limit), PL (plastic limit), w (water content), and e (void ratio). A high LL or PL in a soil is indicative of a more "clay-like" or "plastic" behavior. In general, if the natural water content, w , is closer to PL , the clay may be expected to be stiff, overconsolidated, and have a high undrained shear strength; this usually (but not always) means that the drained condition may be more critical (with respect to the overall stability and the passive resistance of the bearing stratum in a sheet pile problem). On the other hand, if w is closer to LL , the clay may be expected to be soft (Table 3-4), normally consolidated, and have a low, undrained shear strength; and this usually means that the undrained condition will be more critical.

Table 3-1
Granular Soil Properties (after Teng 1962)

Compactness	Relative Density (%)	SPT N (blows per ft)	Angle of Internal Friction (deg)	Unit Weight	
				Moist (pcf)	Submerged (pcf)
Very Loose	0-15	0-4	<28	<100	<60
Loose	16-35	5-10	28-30	95-125	55-65
Medium	36-65	11-30	31-36	110-130	60-70
Dense	66-85	31-50	37-41	110-140	65-85
Very Dense	86-100	>51	>41	>130	>75

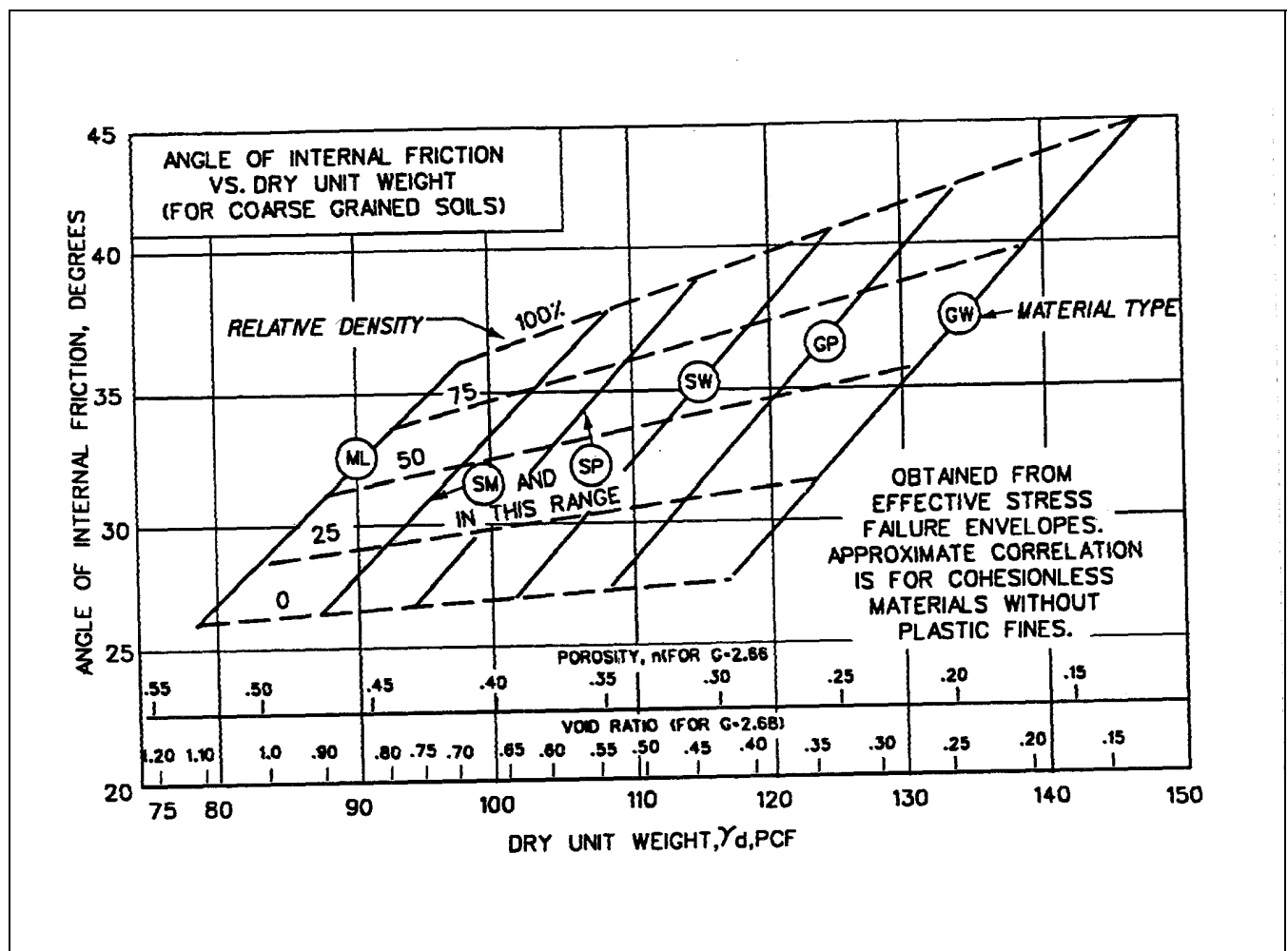


Figure 3-1. Cohesionless Soil Properties (after U.S. Department of the Navy 1971)

Table 3-2
Ratio of ϕ/δ (After Allen, Duncan, and Snacio 1988)

Soil Type	Steel	Wood	Concrete
Sand	$\delta/\phi = 0.54$	$\delta/\phi = 0.76$	$\delta/\phi = 0.76$
Silt & Clay	$\delta/\phi = 0.54$	$\delta/\phi = 0.55$	$\delta/\phi = 0.50$

Table 3-3
Values of δ for Various Interfaces
(after U.S. Department of the Navy 1982)

Soil Type	δ (deg)
(a) Steel sheet piles	
Clean gravel, gravel sand mixtures, well-graded rockfill with spalls	22
Clean sand, silty sand-gravel mixture, single-size hard rockfill	17
Silty sand, gravel or sand mixed with silt or clay	14
Fine sandy silt, nonplastic silt	11
(b) Concrete sheet piles	
Clean gravel, gravel sand mixtures, well-graded rockfill with spalls	22-26
Clean sand, silty sand-gravel mixture, single-size hard rockfill	17-22
Silty sand, gravel or sand mixed with silt or clay	17
Fine sandy silt, nonplastic silt	14

Table 3-4
Correlation of Undrained Shear Strength of Clay ($q_u=2c$)

Consistency	q_u (psf)	SPT (blows/ft)	Saturated Unit Weight (psf)
Very Soft	0-500	0-2	<100-110
Soft	500-1,000	3-4	100-120
Medium	1,000-2,000	5-8	110-125
Stiff	2,000-4,000	9-16	115-130
Very Stiff	4,000-8,000	16-32	120-140
Hard	>8,000	>32	>130

(5) Since an undrained condition may be expected to occur under "fast" loading in the field, it represents a "short-term" condition; in time, drainage will occur, and the drained strength will govern (the "long-term" condition). To model these conditions in the laboratory, three types of tests are generally used; unconsolidated undrained (Q or UU), consolidated undrained (R or CU), and consolidated drained (S or CD). Undrained shear strength in the laboratory is determined from either Q or R tests and drained shear strength is established from S tests or from consolidated undrained tests with pore pressure measurements (\bar{R}).

(6) The undrained shear strength, S_u , of a normally consolidated clay is usually expressed by only a cohesion intercept; and it is labeled c_u to indicate that ϕ was taken as zero. c_u decreases dramatically with water content; therefore, in design it is common to consider the fully saturated condition even if a clay is partly saturated in the field. Typical undrained shear strength values are presented in Table 3-4. S_u increases with depth (or effective stress) and this is commonly expressed with the ratio " S_u/p " (p denotes the effective vertical stress). This ratio correlates roughly with plasticity index and overconsolidation ratio (Figures 3-2, 3-3, respectively). The undrained shear strength of many overconsolidated soils is further complicated due to the presence of fissures; this leads to a lower field strength than tests on small laboratory samples indicate.

(7) The drained shear strength of normally consolidated clays is similar to that of loose sands ($c' = 0$), except that ϕ is generally lower. An empirical correlation of the effective angle of internal friction, ϕ' , with plasticity index for normally consolidated clays is shown in Figure 3-4. The drained shear strength of overconsolidated clays is similar to that of dense sands (again with lower ϕ'), where there is a peak strength ($c' \neq 0$) and a "residual" shear strength ($c' = 0$).

(8) The general approach in solving problems involving clay is that, unless the choice is obvious, both undrained and drained conditions are analyzed separately. The more critical condition governs the design. Total stresses are used in an analysis with undrained shear strength (since pore pressures are "included" in the undrained shear strength) and effective stresses in a drained case; thus such analyses are usually called total and effective stress analyses, respectively.

(9) At low stress levels, such as near the top of a wall, the undrained strength is greater than the drained

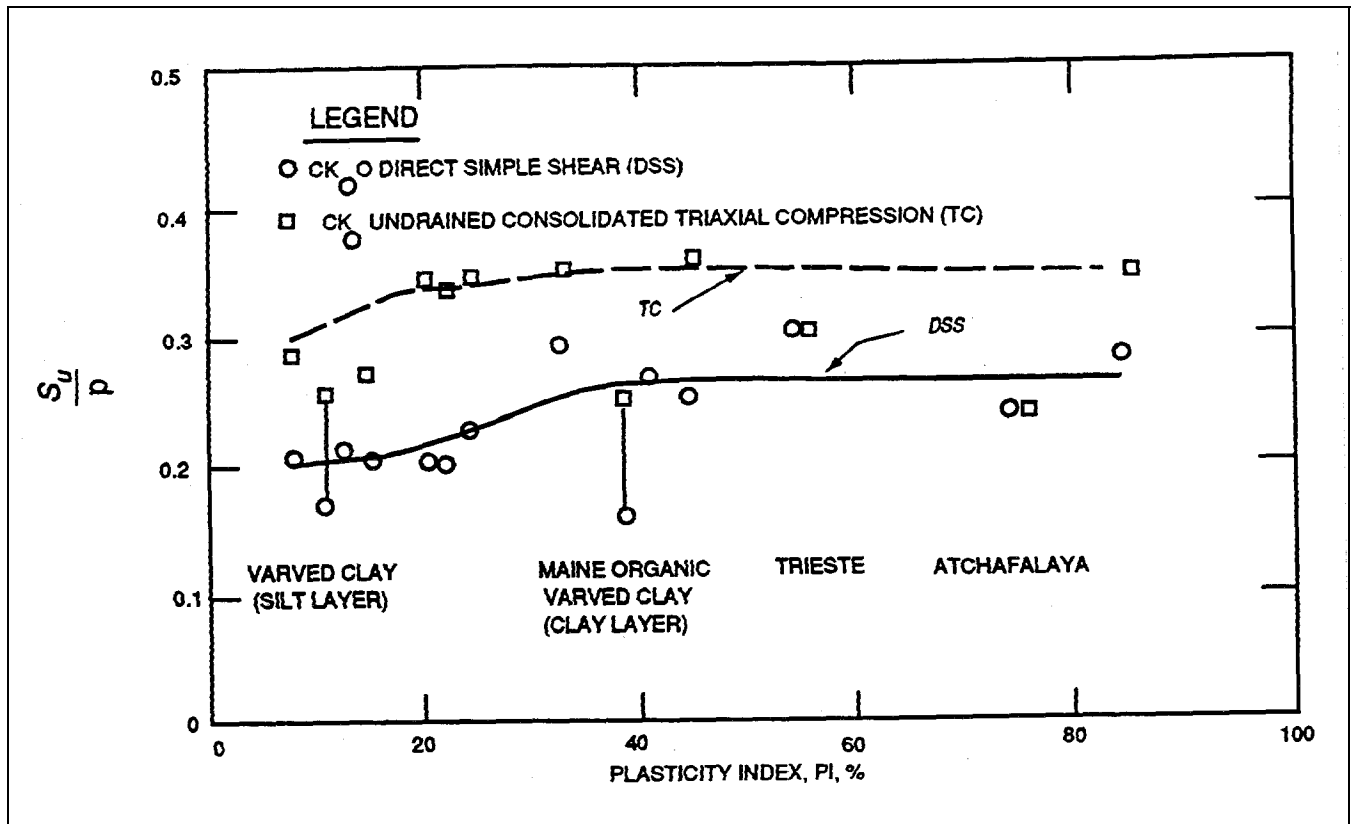


Figure 3-2. Relationship between the ratio S_u/p and plasticity index for normally consolidated clays (after Gardner 1977)

strength due to the generation of negative pore pressures which can dissipate with time. Such negative pore pressures allow steep temporary cuts to be made in clay soils. Active earth pressures calculated using undrained parameters are minimum (sometimes negative) values that may be unconservative for design. They should be used, however, to calculate crack depths when checking the case of a water-filled crack.

(10) At high stress levels, such as below the base of a high wall, the undrained strength is lower than the drained strength due to generation of positive pore pressures during shear. Consequently, the mass stability of walls on fine-grain foundations should be checked using both drained and undrained strengths.

(11) Certain materials such as clay shales exhibit greatly reduced shear strength once shearing has initiated. For walls founded on such materials, sliding analyses should include a check using residual shear strengths.

3-4. In Situ Testing of Foundation Materials

a. Advantages. For designs involving coarse-grain foundation materials, undisturbed sampling is usually impractical and in situ testing is the only way to obtain an estimate of material properties other than pure assumption. Even where undisturbed samples can be obtained, the use of in situ methods to supplement conventional tests may provide several advantages: lower costs, testing of a greater volume of material, and testing at the in situ stress state. Although numerous types of in situ tests have been devised, those most currently applicable to wall design are the SPT, the cone penetration test (CPT), and the pressuremeter test (PMT).

b. Standard penetration test. The SPT (ASTM D-1586 (1984)) is routinely used to estimate the relative density and friction angle of sands using empirical correlations. To minimize effects of overburden stress, the penetration resistance, or N value (blows per foot), is usually corrected to an effective vertical overburden

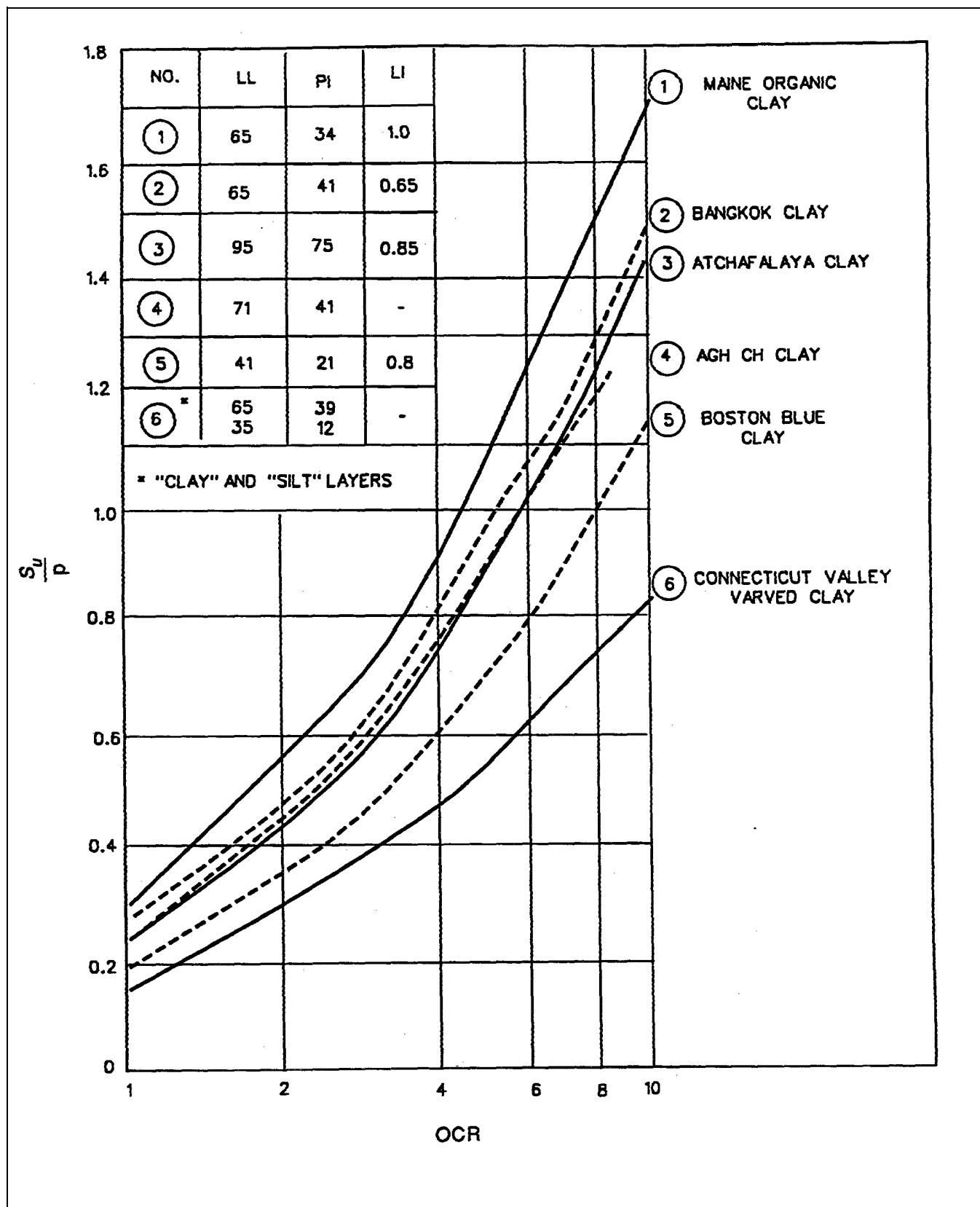


Figure 3-3. Undrained strength ratio versus over-consolidation ratio (after Ladd et al. 1977)

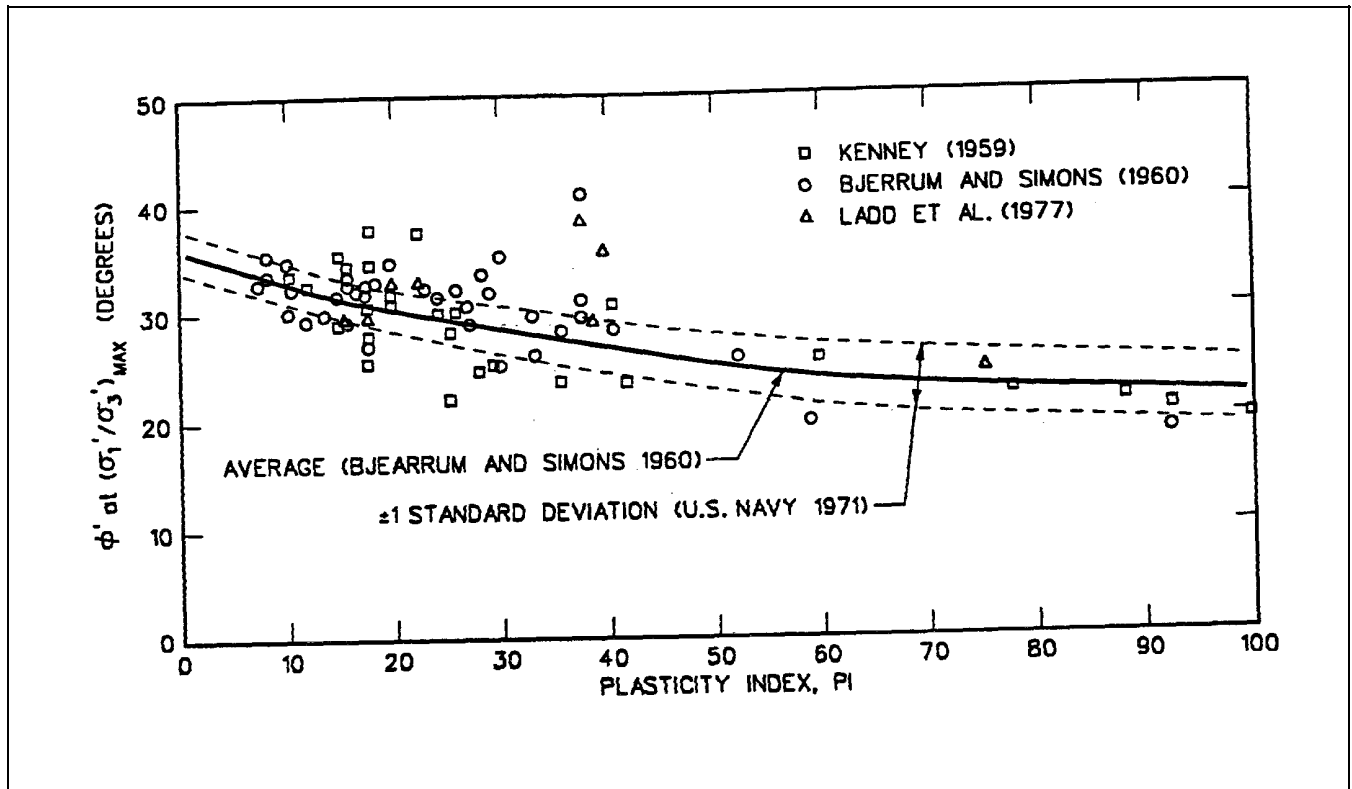


Figure 3-4. Empirical correlation between friction angle and PI from triaxial tests on normally consolidated clays

stress of 1 ton per square foot using an equation of the form:

$$N' = C_N N \quad (3-1)$$

where

N' = corrected resistance

C_N = correction factor

N = measured resistance

Table 3-5 and Figure 3-5 summarize the some most commonly proposed values for C_N . Whitman and Liao (1984) developed the following expression for C_N :

$$C_N = \sqrt{\frac{1}{\sigma'_{vo}}} \quad (3-2)$$

where effective stress due to overburden, σ'_{vo} , is expressed in tons per square foot. The drained friction angle ϕ' can be estimated from N' using Figure 3-6. The

relative density of normally consolidated sands can be estimated from the correlation obtained by Marcuse and Bieganski (1977):

$$D_r = 11.7 + 0.76[|222(N) + 1600 - 53(p'_{vo}) - 50(C_u)^2|]^{1/2} \quad (3-3)$$

where

p'_{vo} = effective overburden pressure in pounds per square inch

C_u = coefficient of uniformity (D_{60}/D_{10})

Correlations have also been proposed between the SPT and the undrained strength of clays (see Table 3-4). However, these are generally unreliable and should be used for very preliminary studies only and for checking the reasonableness of SPT and lab data.

c. *Cone penetration test.* The CPT (ASTM D 3441-79 (1986a)) is widely used in Europe and is gaining

Table 3-5
SPT Correction to 1 tsf (2 ksf)

Effective Overburden Stress kips/sq ft	Correction factor C_N		
	Seed, Arango, and Chan (1975)	Peck and Bazaraa (1969)	Peck, Hanson, and Thornburn (1974)
0.20	2.25	2.86	
0.40	1.87	2.22	1.54
0.60	1.65	1.82	1.40
0.80	1.50	1.54	1.31
1.00	1.38	1.33	1.23
1.20	1.28	1.18	1.17
1.40	1.19	1.05	1.12
1.60	1.12	0.99	1.08
1.80	1.06	0.96	1.04
2.00	1.00	0.94	1.00
2.20	0.95	0.92	0.97
2.40	0.90	0.90	0.94
2.60	0.86	0.88	0.91
2.80	0.82	0.86	0.89
3.00	0.78	0.84	0.87
3.20	0.74	0.82	0.84
3.40	0.71	0.81	0.82
3.60	0.68	0.79	0.81
3.80	0.65	0.78	0.79
4.00	0.62	0.76	0.77
4.20	0.60	0.75	0.75
4.40	0.57	0.73	0.74
4.60	0.55	0.72	0.72
4.80	0.52	0.71	0.71
5.00	0.50	0.70	0.70

considerable acceptance in the United States. The interpretation of the test is described by Robertson and Campanella (1983). For coarse-grain soils, the cone resistance q_c has been empirically correlated with standard penetration resistance (N value). The ratio (q_c/N) is typically in the range of 2 to 6 and is related to medium grain size (Figure 3-7). The undrained strength of fine-grain soils may be estimated by a modification of bearing capacity theory:

$$s_u = \frac{q_c - p_o}{N_k} \quad (3-4)$$

where

p_o = the in situ total overburden pressure

N_k = empirical cone factor typically in the range of 10 to 20

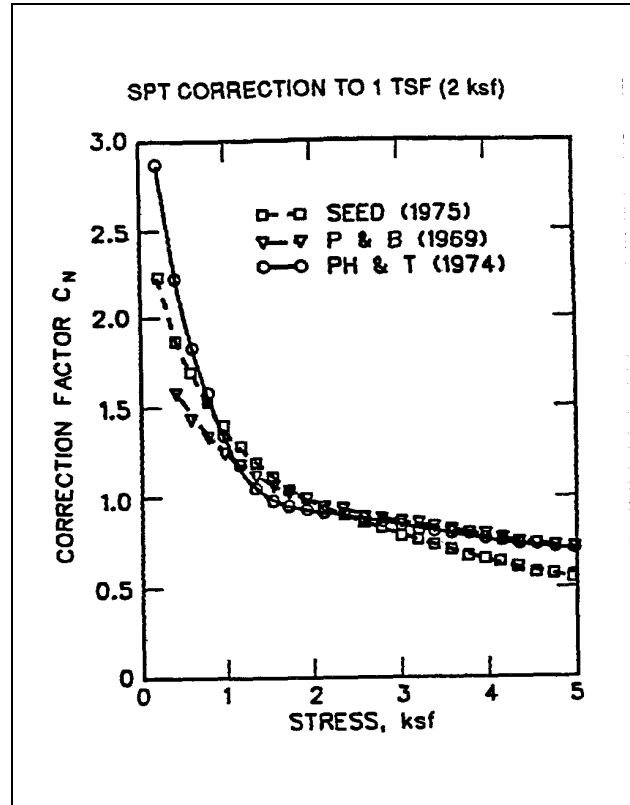


Figure 3-5. SPT correction to 1 tsf

The N_k value should be based on local experience and correlation to laboratory tests. Cone penetration tests also may be used to infer soil classification to supplement physical sampling. Figure 3-8 indicates probable soil type as a function of cone resistance and friction ratio. Cone penetration tests may produce erratic results in gravelly soils.

d. Pressuremeter test. The PMT also originated in Europe. Its use and interpretation are discussed by Baguelin, Jezequel, and Shields (1978). Test results are normally used to directly calculate bearing capacity and settlements, but the test can be used to estimate strength parameters. The undrained strength of fine-grain materials is given by:

$$s_u = \frac{p_1 - p_{ho}}{2K_b} \quad (3-5)$$

where

p_1 = limit pressure

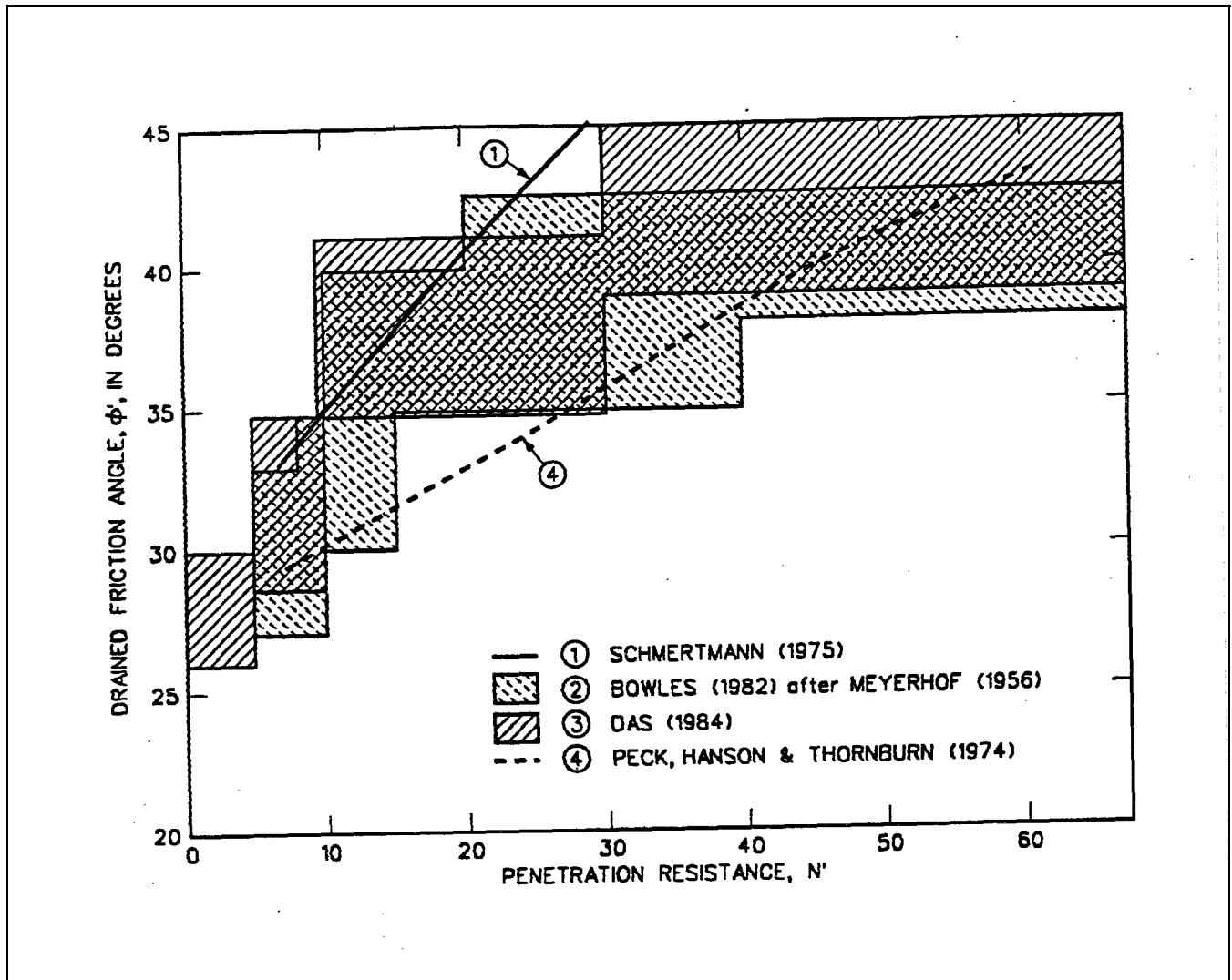


Figure 3-6. Correlations between SPT results and shear strength of granular materials

p_{ho}' = effective at-rest horizontal pressure

K_b = a coefficient typically in the range of 2.5 to 3.5 for most clays

Again, correlation with laboratory tests and local experience is recommended.

3-5. Design Strength Selection

As soils are heterogenous (or random) materials, strength tests invariably exhibit scattered results. The

guidance contained in EM 1110-2-1902 regarding the selection of design strengths at or below the thirty-third percentile of the test results is also applicable to walls. For small projects, conservative selection of design strengths near the lower bound of plausible values may be more cost-effective than performing additional tests. Where expected values of drained strengths (ϕ values) are estimated from correlations, tables, and/or experience, a design strength of 90 percent of the expected (most likely) value will usually be sufficiently conservative.

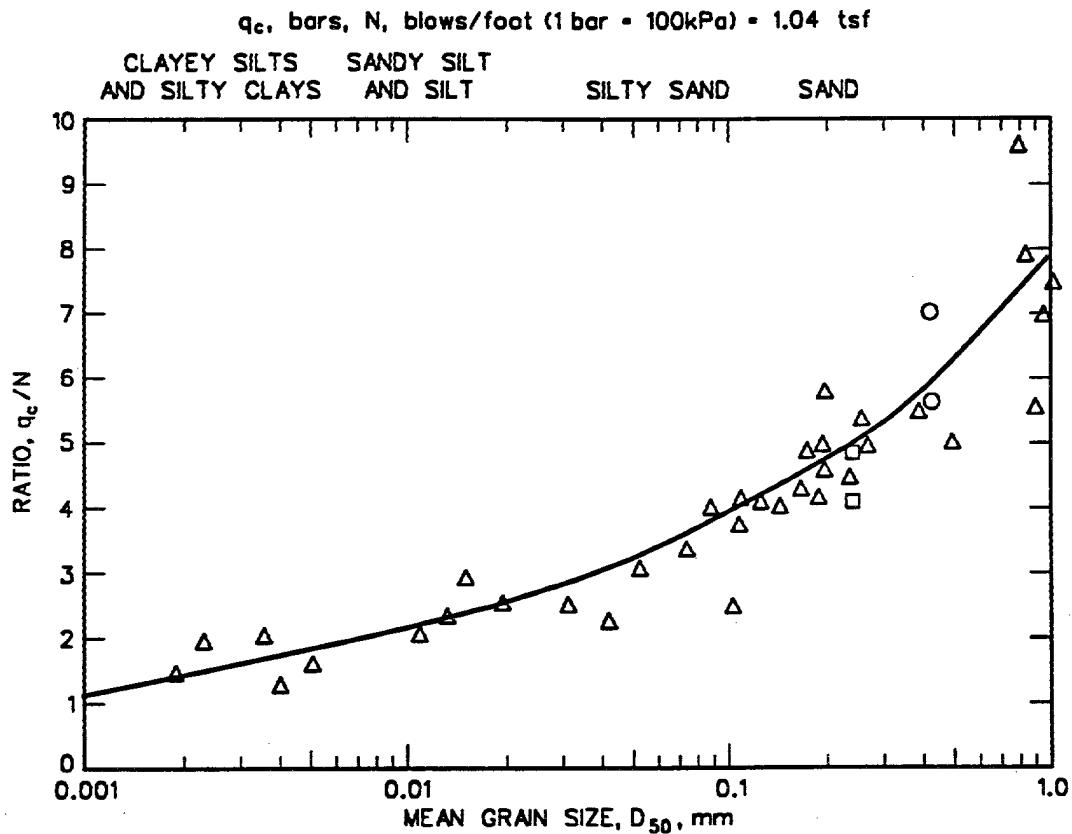


Figure 3-7. Correlation between grain size and the ratio of cone bearing and STP resistance (after Robertson and Campanella 1983)

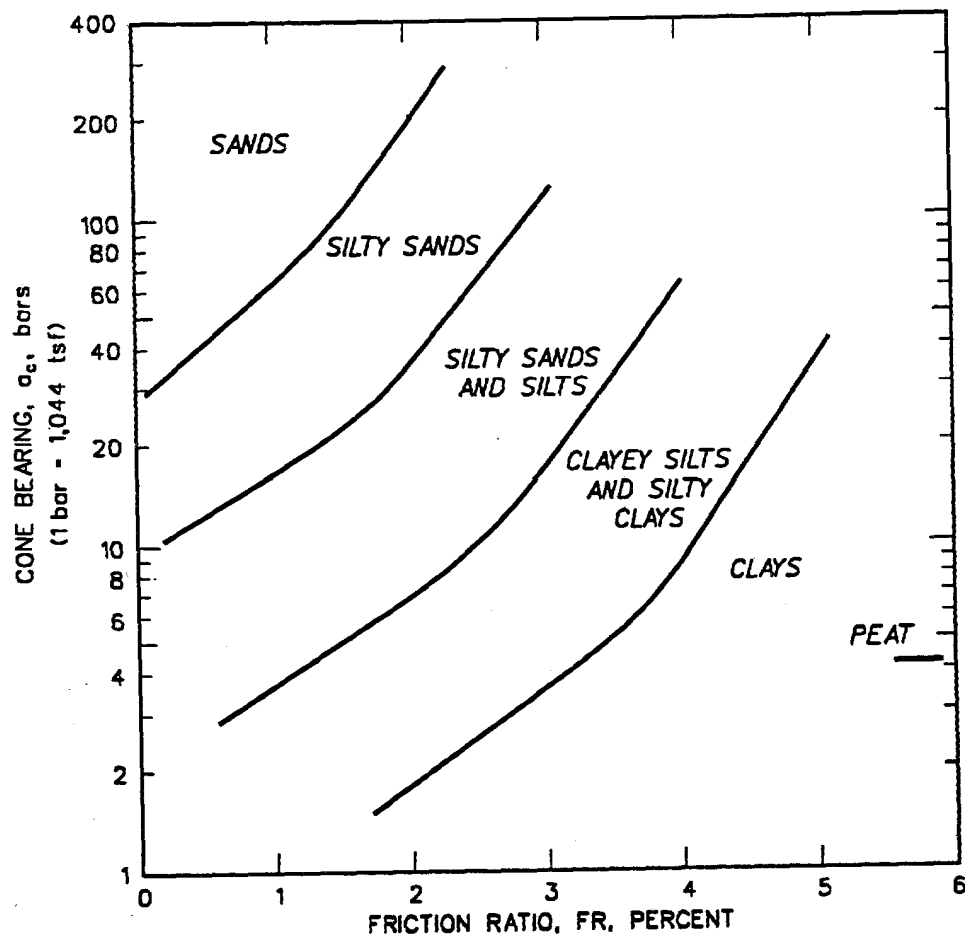


Figure 3-8. Soil classification from cone penetrometer (after Robertson and Campanella 1983)